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State of the Art in the Design of Coastal Structures

by

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Dansk Vandbygningsteknisk Selskabs seminar om
"Nyere metoder til projektering af moler"
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State of the Art in the Design of Coastal Structures

Contributions from MAST, PIANC and some national projects

by

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Contents

1 Introduction

Coastal structures are used in coastal defence schemes with the objective of preventing shoreline erosion and flooding of the hinterland. Other objectives are sheltering of harbour basins and entrances against waves, stabilization of navigation channels at inlets, and protection of water intakes and outfalls. An overview is given in Fig. 1.

Type of structure	Objective	Principal function
Sea dike	Prevent or alleviate flooding by the sea of lowlying land areas.	Separation of shore line from hinterland by a high impermeable structure.
Sea wall	Protect the beach against erosion and alleviate flooding.	Reinforcement of some part of the beach profile.
Groin	Protect the beach against erosion.	Reduction of longshore transport of beach material.
Detached breakwater		Reduction of wave heights in the lee of the breakwater and reduction of longshore transport of beach material.
Reef breakwater		Reduction of wave heights at the shore.
Submerged sill		Retard offshore movement of beach material.
Beach drain		Accumulation of beach material in the drained part of the beach.
Beach nourishment		Artificial infill of beach material to be eroded by waves and currents as compensation for lack of natural supply.
Breakwater	Shelter harbor basins, harbor entrances and water intakes against waves (and currents).	Dissipation of wave energy and/or reflection of wave energy back into the sea.
Jetty	Stabilize navigation channels at river mouths and tidal inlets.	Confine streams and tidal flow. Protection against storm water and cross currents.
Training walls	Prevent unwanted sedimentation or erosion. Protect mooring against currents.	Direct natural or man-made current flow by forcing of water movement along the structure.
Storm surge barrier (barrage)	Protect estuaries against storm surge.	Separation of estuary from the sea by movable locks/gates.
Pipeline Outfall	Transport of fluids.	Gravity based stability.
Pile structure	Provide deck space for traffic, pipelines etc. Provide moorings.	Transfer of deck load forces to the sea bed.
Scour (sea bed) protection	Protect coastal structures against instability caused by sea bed scour.	Provision of resistance against erosion caused by waves and current.

Fig. 1. Types and function of coastal structures (prepared for the Coastal Engineering Manual, U.S. Army Corps of Engineers).

In designing structures optimization is wanted, both with respect to function and to economy seen over a structure service life time. Functional design is still very difficult when it involves structure-sediment interactions as for example for groynes in coastal protection schemes. As to optimal design of the structure itself it can be done today for smaller standard structures solely by the use of desk tools, provided that the sea state conditions are known. However, for large and non–standard structures it is necessary to spend quite a lot of money and time on model tests which for some structures include the use of large scale test facilities. The art do not allow conceptual design just based on available formulae and computer programs. The available sets of tools are not of generic nature as they do not provide the freedom of designing coastal structures which deviate significantly from the well proven and documented traditional design concepts.

Despite this not fully satisfactory situation the paper does reflect the significant progress which has been made within the last ten to fifteen years where quite a lot of national and international research projects and special working groups related to coastal structures have been running. Amongst the significant international activities are the European Union MARINE SCIENCE and TECHNOLOGY research program MAST and the Working Groups of PIANC.

The background of the many recent research activities is twofold:

- A series of very serious damages to newly designed major breakwaters appeared, clearly demonstrating large uncertainties in the design tools and even lack of knowledge about important failure modes.
- It became more and more incomprehensible why coastal structures could not be designed with a confidence which matches other civil engineering structures.

The last point involves the fact that only in very few cases the design was based on a rational safety implementation analysis including consideration of major uncertainties, f.ex. on the wave climate estimates.

The paper focuses on safety implementating as well as on results and tools related to the conceptual design of breakwater cross sections. Lay-out design is not discussed because it is too site specific. However, it should not be forgotten that the lay-out is, in most cases, even more important for satisfactory function and economy. Fig. 2 shows examples of the three main types of breakwaters.

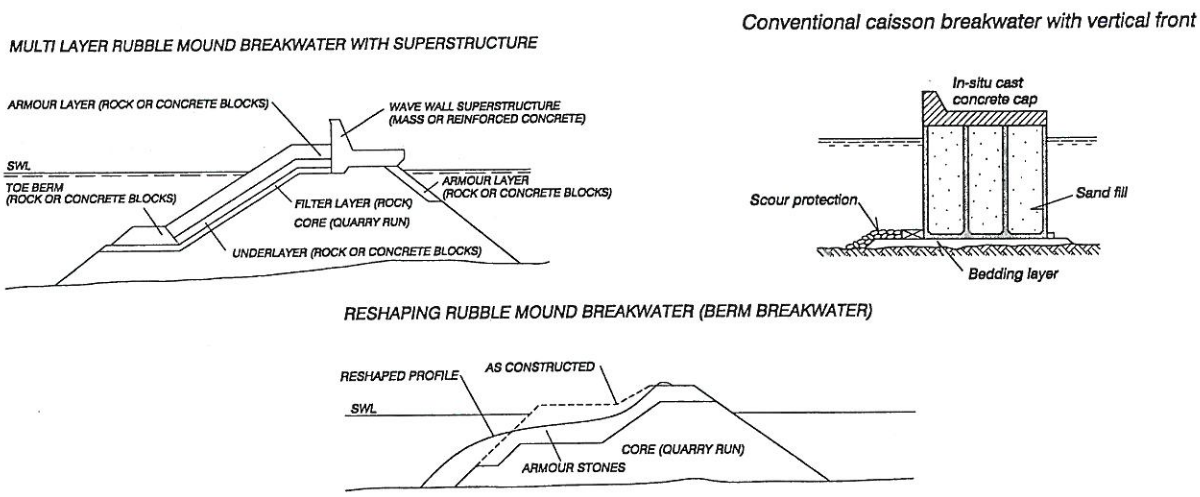


Fig. 2. Main types of breakwaters.

2 Design procedure

a **Functional requirements** and **structure life time** are first to be specified. For breakwater cross sections it involves requirements for accessroads, installations, moorings, etc., and related to this also a definition of allowable deformations, wave overtopping, wave transmission, and wave reflection. The last three points constitute the acceptable **hydraulic responses.**

b **Local long-term and short-term wave/water level statistics** have to be established.

c **Alternative structure geometries** which meet the functional requirements are sketched.

d **All possible failure modes** related to the alternatives are identified.

e **Safety levels** related to the failure modes are defined.

f **Method of implementation of safety** is selected.

g **Conceptual design of the alternatives** is performed in which all failure modes are investigated and the overall stability and the stability of structural members are ensured to meet the target safety levels. This includes assessment of the long term durability of the structural parts.

h **Selection of preferred design**, based on economical optimization.

The contributions from MAST projects and PIANC–working groups are related mainly to items a, d, e, f, and g.

3 Important recent projects

The first large European project on coastal structures was the 1990–92 MAST project:

(i) **Coastal Structures.** MAST project G 6 – S. Contract 0032, coordinated by HR Wallingford.

This project had an ice–breaker effect in creating strong links between European research groups, and it formed the basis for a series of MAST projects:

(ii) **Monolithic coastal structures** MAS2–CT92–0047, coordinated by H. Oumeraci, Tech. Univ. of Braunschweig.

(iii) **Rubble mound breakwater failure modes** MAS2–CT92–0042, coordinated by H.F. Burcharth, Aalborg Univ.

(iv) **Full-scale dynamic load monitoring of rubble mound breakwaters** MAS2–CT92–0023–C, coordinated by J. De Rouck, Univ. of Ghent.

(v) **Berm breakwater structures** MAS2–CT94–0087, coordinated by J. Juhl, Danish Hydraulic Institute.

(vi) **Reflection of waves from natural man-made coastal structures** MAS2–CT92–0030–C, coordinated by M. Losada, Cantabria Univ.

Further the following MAST 3 projects are ongoing more or less as a continuation of two of the above mentioned projects:

(vii) **Probabilistic design tools for vertical breakwaters** MAS3–CT92–0041, coordinated by H. Oumeraci, Tech. Univ. of Braunschweig.

(viii) **The optimization of crest level design of sloping coastal structures through prototype monitoring and modelling** MAS3, PL 961158, coordinated by J. De Rouck, Univ. of Ghent.

Another partly EU sponsored project is the combined LIP–MAST2–(MCS)–TAW project:

(ix) **Wave overtopping and loads on caisson breakwaters under three-dimensional sea states** coordinated by L. Franco Politecnico di Milano, H.F. Burcharth Aalborg Univ., and J.W. van der Meer Delft Hydraulics.

This paper concerns only the effect of non–breaking waves for which reason the following project, sponsored by The Danish Technical Research Council (STVF) was undertaken at Aalborg University:

(x) Wave loads and overtopping on caisson breakwaters from three–dimensional breaking waves.

Like many other European national sponsor organizations STVF also sponsored, through its MARIN TEKNIK programme a number of research projects related to coastal structures. Many of the projects were closely coordinated with the MAST and PIANC activities.

Examples are the following projects:

(xi) Wave forces and overtopping of crown walls of rubble mound breakwaters, J. Pedersen, Aalborg Univ.

(xii) Application of reliability in breakwater design, E. Christiani, Aalborg Univ.

(xiii) Scour around breakwater roundheads, M. Sumer and J. Fredsøe, Technical Univ. of Denmark.

The following two PIANC working group projects have contributed to the development of breakwater design methods by introducing the fully probabilistic approach.

(xiv) **Analysis of rubble mound breakwaters** PIANC PTC II Working Group 12, chairman J.D. Mettam, Scott Wilson Kirkpatrick.

(xv) **Breakwaters with vertical and inclined concrete walls** PIANC PTC II Working Group 28, chairman H.F. Burcharth, Aalborg University.

The MAST projects are all very well documented in project reports and workshop proceedings. The PIANC Working Group results are published in the PIANC–Bulletins and in special Working Group reports. The main results of national programmes are published in conference proceedings and/or in journals. A systematic presentation of the MAST projects will not be given here. Reference is made to Oumeraci et. al (1995) in which the MAST 2 projects (ii) – (vi) are discussed.

4 Hydraulic responses

4.1 Run-up

Run-up caused by long–crested head-on waves on most types of rubble mound structures can be predicted quite well. This includes straight slopes and to some extent bermed slopes armoured with rock or concrete armour units (e.g. cubes, Antifer blocks with and without holes, Tetrapods, Dolosse, Accropodes). The information is scattered. Main references are van der Meer (1993) and CUR (1994). Additional references from project (ii) are Berenguer et al. (1994 a and b) and Burcharth et al (1995 b). Most results rely on small scale model tests. Some concern about significant scale effect initiated by the above mentioned project (viii).

4.2 Wave transmission

Wave transmission related to low-crest and submerged rubble structures can be predicted with reasonable accuracy. Main reference is van der Meer (1993). Information on wave transmission over vertical wall structures is given in Goda (1985).

4.3 Wave overtopping

For rubble mound structures without crown walls there is still insufficient information for a more accurate estimation of overtopping in general, see van der Meer (1993), Allsop et al. (1992), and Canel et al. (1992). The latter reference includes the effect of short-crested waves.

For rubble mound breakwaters with crown walls quite a lot of information is available, see f.ex. references in CUR (1994) and van der Meer (1993). The most recent results come from a well documented comprehensive parametric model test study, Pedersen (1996), resulting in a formula for the mean overtopping discharge as function of the breakwater geometry and the sea state. The formula has only one fitting parameter and is valid for head-on long–crested waves.

Overtopping of vertical wall structures has been investigated by Goda (1985) and in a number of recent projects among which are projects (ii), (ix), (x). This includes caisson breakwaters with different crest geometry and with and without perforated front tested in 2–D and 3–D models. The results comprise the effect of oblique short–crested non–breaking waves, Franco et al. (1997), and of breaking (top spilling) waves, Groenbech et al. (1997). The first reference gives, besides the mean discharge, also recommendations of acceptable levels of overtopping. Fig. 3 shows tested cross sections.

The lateral distribution of the overtopping water behind the breakwater front has not been investigated systematically but only for specific cases, see f.ex. Burcharth et al. (1995 a). However, in project (viii) there will be performed a comparative analysis of the distribution between prototype measurements at Zeebrugge and small scale model measurements. This will reveal quantitative information on scale effects.

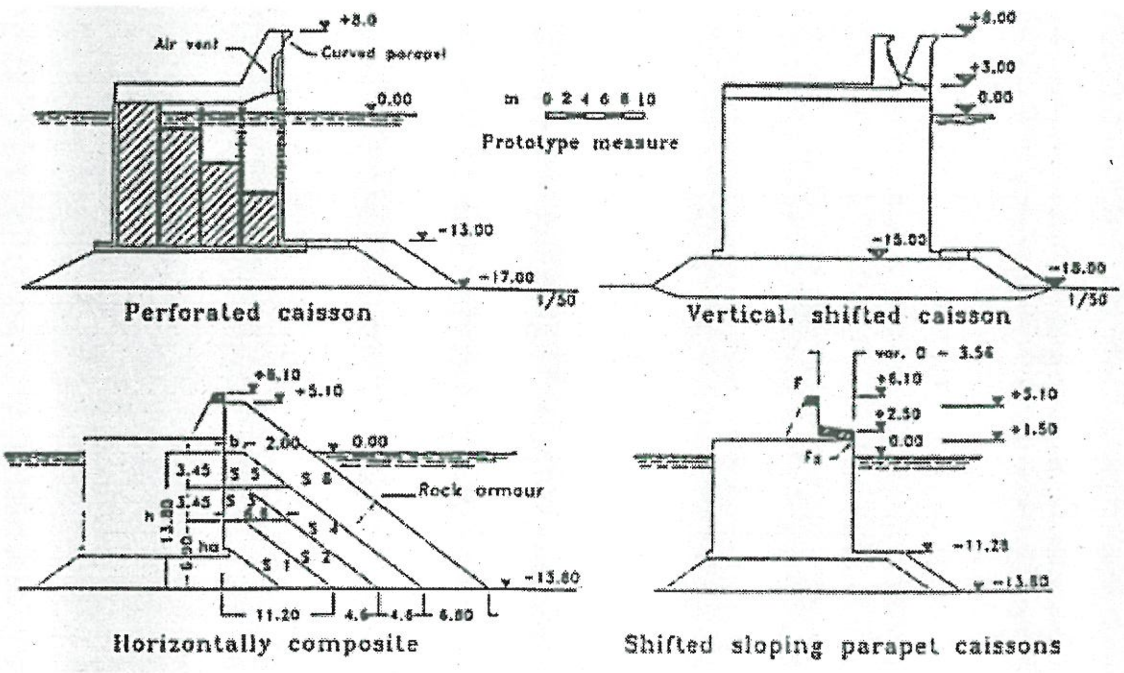


Fig. 3. Cross sections tested in the LIP-MAST 2-TAW project (Franco et al. (1992)).

4.4 Wave reflection

For sloping structures the information is still limited and scattered, but generally it is possible to get some reasonable estimates on the reflection coefficients for head-on long–crested waves, see Allsop et al. (1988), Canel et al. (1992), and CUR (1994). For oblique long–crested waves see Benoit et al. (1994), for detached breakwater field measurements Davidson et al. (1994), and for berm breakwaters Mansard (1991).

For vertical wall structures reflection coefficients for impermeable and permeable fronts were studied in project (ix), Helm–Petersen (1994 and 1995), and for porous (stone–filled) structures at Aalborg University, Helm–Petersen (1996). Both projects included the effect of oblique short–crested waves. Results from 2–D tests performed in project (ii) with perforated screens are given in Allsop et al. (1994).

4.5 Internal porous flow and pore pressure built op

Wave induced non–stationary flow in rubble mound breakwaters and in high mounds and bedding layers under caisson structures as well as in the subsoil is of importance both for the soil mechanics stability of the structures, the soil strength and the uplift pressures on bottom slabs. Very complicated scaling laws and soil strength-dependence on stress level make scale model tests very questionable for which reason numerical modelling is the solution. Partly sponsored by project (i), the problem about scaling of non-stationary flow in coarse stone materials (like quarry rubbles) was solved, partly by experiments, van Gent (1994), Burcharth et al. (1995).

Several numerical models for wave induced flow have been developed in the course of projects (i), (iv), and others, see f.ex. van Gent (1994). The models are not yet entirely based on basic physics but must be calibrated against small scale model results. The latter involves yet unsolved scale effect problems.

Pore pressure built–up under cyclic wave loading is related to porous flow but dealt with experimentally in cyclic loaded triaxial apparatus in order to develop constitutive equations for subsoil materials. Based on experience from offshore platform designs and on extensive advanced laboratory soil tests within project (ii) some guidelines for design of foundation under caisson structures have been presented. The work continues in project (vii) and will probably result in sets of constitutive equations (for characteristic soils) necessary for numerical stability calculations. A comprehensive report on basic mechanics problems related to vertical wall structures is published by the Geotechnical Group (coordinated by M. de Groot, Delft Geotechniques) in the proceedings of project (ii). It contains very useful diagrams for prediction of excess pore pressures in sand foundation under cyclic loading, Andersen (1995).

5 Wave loads and related responses

5.1 Scale and model effects

It is necessary to know the simultaneous load distribution on the surface of the breakwater elements in order to calculate the stability, i.e. the pressure on the front and rear faces as well as uplift pressures on bottom slabs must be known. Because field investigations are extremely expensive and time consuming we are forced to use small scale model experiments. This causes serious scaling problems because the effect of breaking waves do not obey the Froude similarity. This is because compressibility related to the enclosed air bubbles and pockets, and the surface tension effects are not accounted for. Moreover, the size of air bubbles is very different in salt and fresh water, the latter being used in the models. As a result it is uncertain how to convert results involving breaking waves from model to prototype except that the Froude–scaling law gives the upper limit of the forces. The problem has been studied by many researchers and was – for vertical face breakwaters – an important part of project (ii). The research continues in project (vii) which includes several prototype investigations. However, in order to reach more precise conversions it is necessary to know the volume and distribution of the enclosed air. The pulsation period of entrapped air pockets is visible in the force history and can be converted by the use of the Mach–Cauchy scaling law.

Wave induced uplift pressures are affected by the permeability of the foundation soil for which reason viscosity plays an important role in converting from model to prototype. In case of high frequency wave impacts also the compressibility has to be considered. For these reasons the Froude–scaling law cannot be applied. The solution will be development of numerical models which – after proper calibration against results from both scale models and prototypes – can handle this highly transient problem. Both projects (vii) and (viii) deal with this task.

So far has – on the safe side – been used a linear variation of the uplift pressure between the recorded pressures of the edges of the foundation slab, and a Froude scale conversion of the results.

In case of caisson breakwaters not exposed to provoked wave breaking Goda (1985) recommends a certain reduction rule based on prototype experience. Indications about the deviation of the uplift pressures from a linear variation can be found in Losada et al. (1993).

5.2 Wave loads on sloping rubble structures

Wave loadings are treated very differently for sloping rubble structures (like armour layers) and monolithic structures (like caissons and wave walls). In the first case the tradition is to avoid speculations about the wave pressures on the rubble units – simply because of the extremely complicated flowfield of breaking waves on a rough porous slope. For such cases the design is based on formulae expressing directly the response in terms of displacements as function of the sea state, see Chapter 6. The wave loadings are simply put away in a black box. Attempts to measure wave forces on individual blocks have been made in projects (i) and (v), mainly with the objective of getting basic information which can explain the surface profile development of berm (reshaping) breakwaters. Simple numerical models for profile development were also developed, but so far it has not been possible to produce generally applicable models.

5.3 Wave loads on monolithic vertical wall structures

For stability calculation of monolithic structures the wave loading must be known. Although the front face geometry of f.ex. a caisson is very simple, the wave pressures are very complicated functions of the sea state. Fig. 4 illustrates typical force histories.

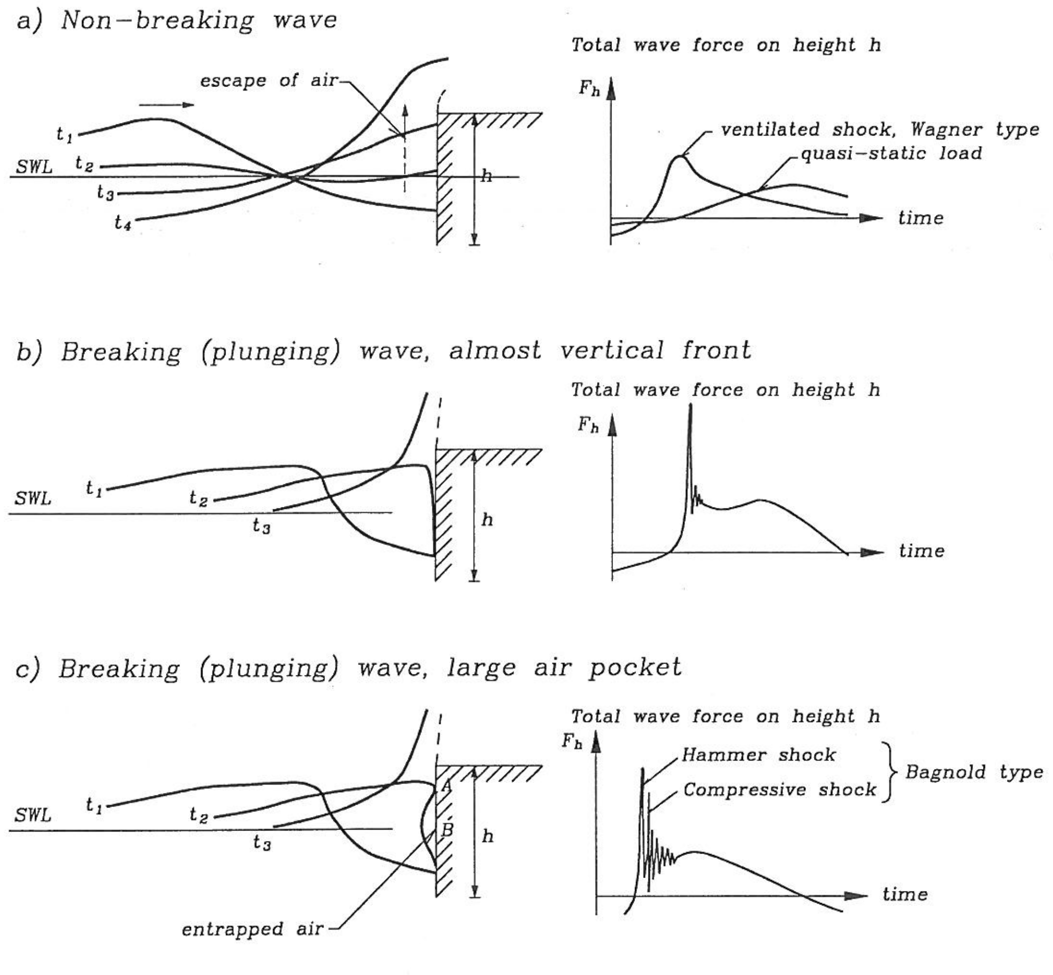


Fig. 4. Illustration of vertical wall wave forces from non–breaking and breaking waves.

The character of the force histories and the pressure distributions have been intensely studied also in large scale facilities at University of Hannover and University of Braunschweig, Oumeraci et al. (1993), and was an important part of project (ii).

Most simple to predict is the loading from non-breaking waves for which case it has been experimentally demonstrated in project (ix) that wave pressures on long structures from oblique short-crested seas, as predicted by Battjes (1982), can be closely estimated by real time calculations, Frigaard et al. (1995).

However, in real storm sea states the waves will break, either by top-spilling as in deep water wave situations, or, in case of shallow water and semi high steep slopes in front of the wall, by plunging. The top-spilling breakers increase the wave load somewhat compared to non-breaking waves and introduce also some higher frequency loadings. It has been demonstrated in project (ii), by comparing with various 2-D model tests at the Danish Hydraulic Institute and Delft Hydraulics, that the Goda formula, which includes the effect of top-spilling wave breaking in real seas, predicts very well the wave loads. The comparison confirmed the deliberately built-in safety margin in the formula. The comparison might however be slightly biased because top-spilling wave breaking in real seas is probably more pronounced than in a wave flume. Project (x) expands the investigations to include the effect of top-spilling breakers on long caisson structures in oblique short-crested seas. Absorption of reflected short-crested waves was used in the project, probably for the first time in a regular testing program. Fig. 5 shows the layout of the model and the position of the wave gauges.

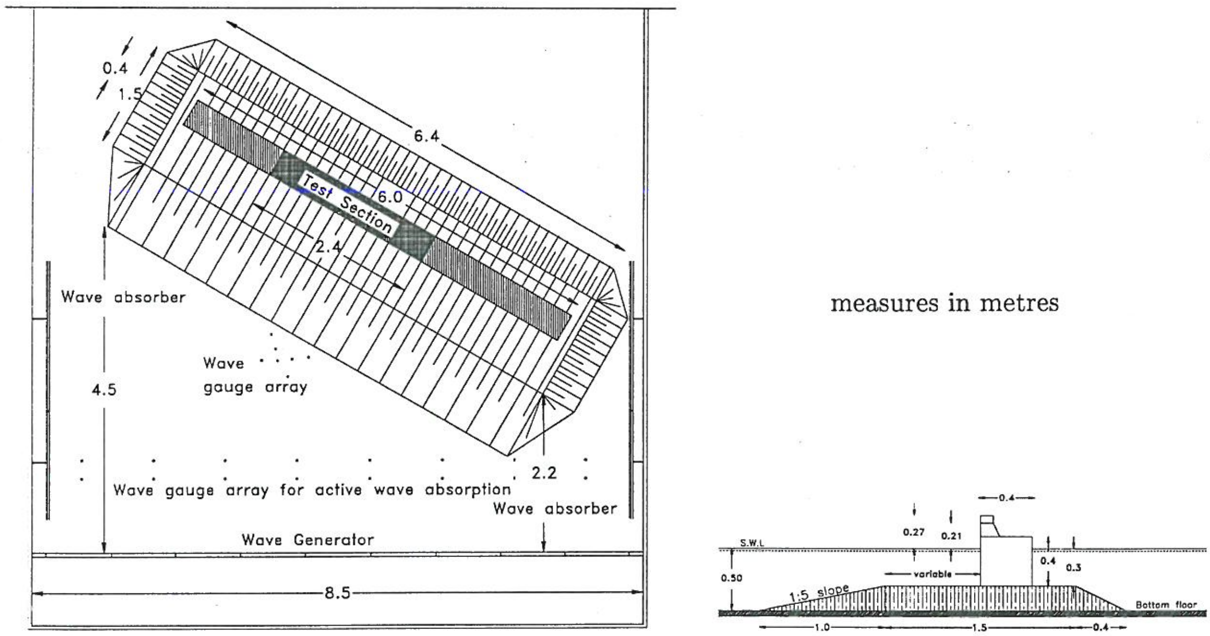


Fig. 5. Test set-up for investigation of wave forces and overtopping in short-crested oblique breaking waves (top spilling). Aalborg University.

Goda (1985) stresses that designs which provoke plunging waves to occur at the structure should be avoided because of the very large impulsive pressures which are generated. Such pressures are responsible for several failures. However, if significant impulsive pressures are to be accounted for, a fully dynamic stability analysis must be performed. This involves the following complications.

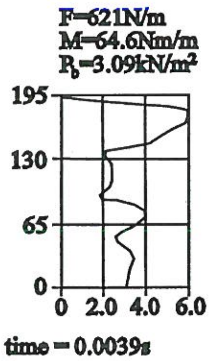
- The loadings are extremely difficult to predict due to scale effects, see section 5.1 and due to large sensitivity to the shapes of the waves and the structure geometry. The latter introduces very large scatter.

- The transmission of the loading to the subsoil and the effect on the subsoil characteristics are very complicated.

Solving these problems is part of projects (ii) and (vii). The latter involves 2–D wave load experiments at HR–Wallingford to determine more narrow bands for occurrence of impulsive pressures and the statistics of the loads, Allsop et al. (1996). The dynamic response in terms of deformation of the subsoil has been analysed, Oumeraci et al. (1994), Pedersen (1994 and 1997), Benassai et al. (1997), using linear elastic models for the soil. From this work it can be estimated when it is necessary to use a dynamic analysis. Ibsen et al. (1997) developed a 2–D method based on plasticity for the prediction of permanent soil/structure deformations due to impulsive loadings. Moreover, within project (vii) very interesting prototype experiments with dynamic loading of Italian caissons are performed by University of Bologna. Further, a spring–dashpot model, Kortenhaus et al. (1995), will be tested against the known behaviour of prototype caissons. Generally, progress is made in establishing operational tools for determination of the dynamic wave–structure–soil interactions, see also Sections 4.5 and 5.1. The Geotechnical Group of project (vii) plays a major role in this.

5.4 Wave loads on crown walls

The simultaneous wave induced pressures on the front face and the uplift pressures must be known in order to analyse the crown wall stability. Fig. 6 shows a typical time history pressure distribution on a crown wall.



Test no. 202
$H_s = 0.173\,m$
$T_P = 2.2\,s$
$A_c = 0.11\,m$

No. pressure cells : 16
Vert. cell spacing : $11\,mm$
Hor. cell spacing : $40\,mm$
Logging frequency : $256\,Hz$

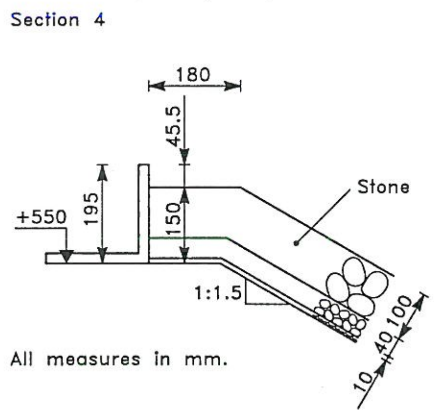


Fig. 6. Example of pressure distribution on crown wall (Pedersen 1996).

A first approach to a realistic model for the pressure distribution over the wave wall front face was presented by Günbak et al. (1983). Pedersen (1996) presented the first systematic parametric model test study of wave forces on crown walls of different heights including influence of water level, berm height and width, slope and armour type. The results comprise the statistics of the horizontal force, the tilting moment and the wall base pressure, thus providing the basis for stability calculations. The uplift pressure distribution is discussed in Section 5.1.

6 Probabilistic failure mode analysis and safety coefficients

6.1 Failure mode overview

Figs. 7 and 8 give an overview of important failure modes related to rubble mound and vertical face monolithic breakwaters. Key references are Ligteringen (1992 and 1997), Oumeraci (1992).

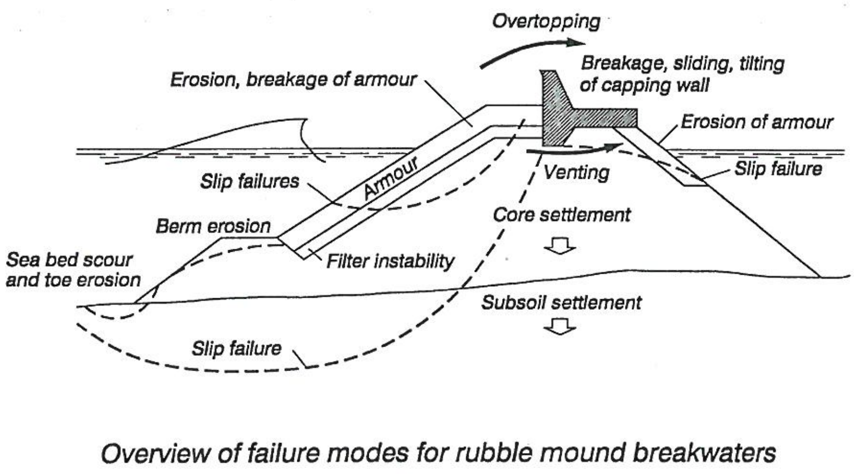


Fig. 7. Rubble mound breakwater failure modes.

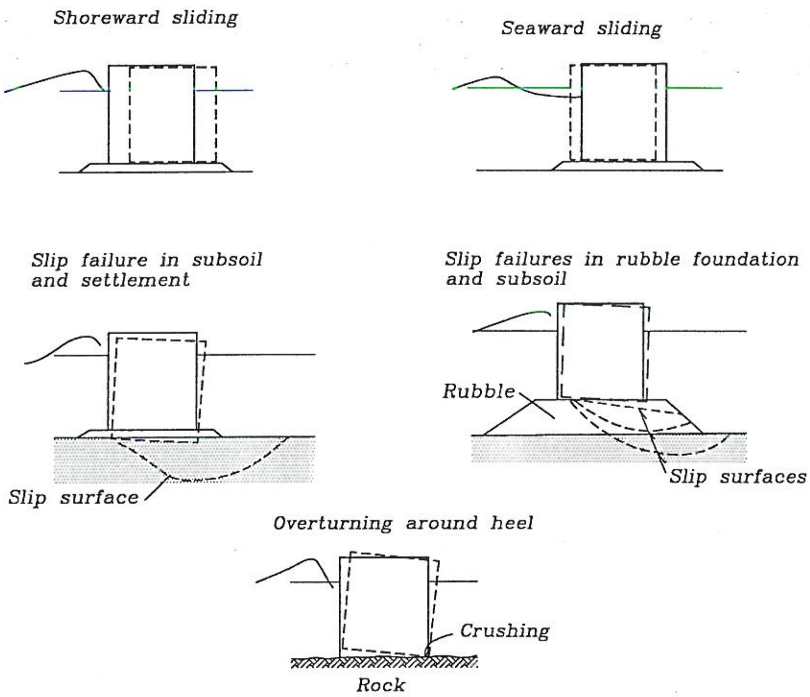


Fig. 8. Important global failure modes for caisson breakwaters.

6.2 The probabilistic approach

The basic principle in reliability analyses is that the probability of damage (usually denoted *failure*) is assessed for each failure mode. On this basis it is possible to estimate the safety or reliability of the whole structure by system analysis.

In each failure mode analysis all load and resistance parameters are treated as stochastic variables. Consequently, it is necessary to introduce the uncertainty of these parameters, e.g. on the sea–state parameters (load) and the soil/strength parameters (resistance). Moreover, the uncertainty on the applied formulae or calculations methods must also be taken into account. The uncertainties on the parameters are given by their probability distributions. As an example is the long term distribution of the max. significant wave heights in storms often given by a Weibull distribution. For most parameters a normal distribution with a certain standard deviation is used. The same holds for uncertainties on formulae. Most often a safety–index method is used for the estimation of the damage probability related to each failure mode, see f.ex. Burcharth (1997).

Related to breakwaters, this approach was first used in a systematic way in the PIANC PTC II Working Group 12, project (xiv), Burcharth (1991), PIANC (1992), and in the MAST 2 project (iii), Soerensen et al. (1995), both for rubble mound structures.

The approach was subsequently used for vertical wall structures in the PIANC PTC II Working Group 28 (project (xv) (to be published ultimo 1997), and it is now a fundamental part of the ongoing MAST 3 project (vii).

6.3 Failure mode partial safety coefficients

The PIANC Working Groups developed on the basis of extensive numerical simulations a system of safety factors for most of the rubble mound breakwater failure modes and for the overall stability failure modes related to caisson breakwaters. The latter is based on static analyses in which wave loads are calculated from the extended Goda formula which includes impulsive pressures, Takahashi et al. (1994). Verification of the use of the coefficients in a fully dynamic analysis will be checked in project (vi) and the system will be extended to include important local structure failure modes for caisson breakwaters.

The PIANC safety factor system is a new and advanced system which provides the safety coefficients corresponding to any wanted safety level (probability of failure) and structure life time, Burcharth (1991), Soerensen et al. (1995). This means that structures, by very simple calculations, can be designed to given safety levels, f.ex. 20% probability of a certain damage within 50 years. Other safety coefficient systems, as f.ex. the Euro Code, do not give coefficients corresponding to prespecified safety levels but only coefficients covering very wide and, for breakwaters, insufficiently specified safety classes. Table 1 contains examples of PIANC safety coefficients. The standard deviations of 0.05 and 0.20 on H_s source data correspond typically to accelerometer buoy recordings and fetch scatter diagram estimates for significant wave heights, respectively, Burcharth (1992).

Table 1. Example of design formulae and related partial safety coefficients γ_Z and γ_H corresponding to various failure probability within T years. (Burcharth 1991, Soeerensen et al. 1995 and 1997).

Armour stability (Hudson formula)

$$0 = \frac{1}{\gamma_Z}\hat{\Delta} D_n (K_D cot\hat{\alpha})^{1/3} - \gamma_H \hat{H}_S^T$$

$\hat{...}$	central estimate (mean value)
K_D	Damage coefficient
α	slope angle
D_n	(volume of armour unit)$^{1/3}$
Δ	$\frac{\rho_s}{\rho_w} - 1$, where ρ_s and ρ_w are block and water mass densities
H_s^T	T-years return period value of significant wave height

Failure probability	coefficient of variation on H_S source data 0.05		0.20	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.7	1.04	2.0	1.00
0.05	1.4	1.06	1.6	1.02
0.10	1.3	1.04	1.4	1.06
0.20	1.2	1.02	1.3	1.00
0.40	1.0	1.08	1.1	1.00

Horisontal sliding of caisson on rubble foundation

$$0 = \frac{1}{\gamma_Z}\hat{f}\left[\hat{F}_G - 0.9\hat{F}_U(\gamma_H \hat{H}_S^T)\right] - 0.9\hat{F}_H(\gamma_H \hat{H}_S^T)$$

$\hat{...}$	central estimate (mean value)
f	friction factor
F_G	gravitational force on caisson
H_s^T	T-years return period value of significant wave height
$F_U\left(\gamma_H H_S^T\right)$	Uplift-pressure in which significant wave height is $\gamma_H \hat{H}_s^T$
$F_H\left(\gamma_H H_S^T\right)$	Horizontal wave load in which significant wave height is $\gamma_H \hat{H}_s^T$

Failure probability	coefficient of variation on H_S source data 0.05		0.20	
P_f	γ_H	γ_Z	γ_H	γ_Z
0.01	1.4	1.7	1.5	1.7
0.05	1.3	1.4	1.4	1.4
0.10	1.3	1.2	1.4	1.3
0.20	1.2	1.2	1.3	1.2
0.40	1.1	1.0	1.1	1.1

Deep water conditions. Model tests not performed.

7 New failure mode analyses and formulae

7.1 Geotechnical failures

A systematic study of slip surface–rupture zone geotechnical failures related to crown walls and caisson structures is presented in Christiani (1997). Fig. 9 shows examples of the investigated failure modes. The relative importance of the various failure modes is evaluated using probabilistic quasi–static stability analyses on typical prototype structures.

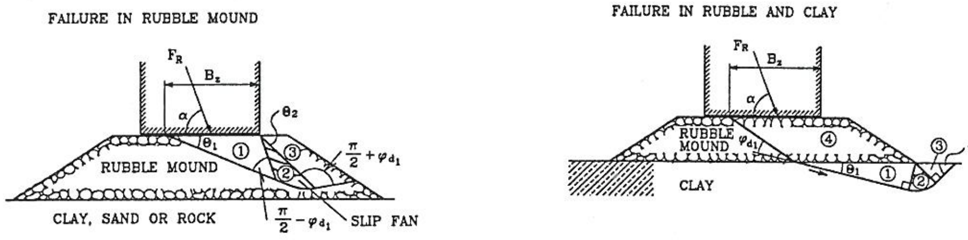


Fig. 9. Examples of geotechnical failure modes (Christiani, 1997).

7.2 Armour layers

Displacement of armour units

References to newer formulae and/or information for estimation of displacement of armour units are:

Rock: Galland (1992), Allsop et al. (1992), van der Meer (1993), CUR (1994), Canel et al. (1992). The latter reference deals with the effect of multidirectional waves.

Accropode: van der Meer (1988), Holtzhausen et al. (1991), Galland (1992), Canel et al. (1992), Kobayashi et al. (1994), Christensen et al. (1995).

Core Loc: Melby et al. (1994), Melby et al. (1997).

Dolosse (includes roundheads and effect of wave walls): Burcharth (1993), Burcharth et al. (1994), Burcharth et al. (1995b).

Tetrapods: van der Meer (1988), Canel et al. (1992), Galland (1992), van Nes (1995).

Cubes: van der Meer (1988), Allsop et al. (1992).

Hollowed cubes and hollowed Antifer cubes include roundheads and effect of wave walls: Galland (1992), Canel et al. (1992), Berenguer et al. (1994a), (1994b), (1995).

Low crest rock breakwaters: Burger (1995), van der Meer et al. (1995).

Rock toe berm–main armour interaction

Project (iii) included a comprehensive experimental study of rock toe berm–main armour interaction in 2–D and 3–D short–crested waves. Conditions for toe failures and the effect on armour stability were identified and a probabilistic model for the interaction was formulated. Lamberti

(1994), Benoit et al. (1996), Donnars et al. (1996), van der Meer et al. (1996), Christiani (1997).

Breakage of slender concrete armour units

Prediction of breakage of slender concrete armour units was for a long period a problem because no formulae for estimation of breakage were available. Recent cooperation within project (iii) between Aalborg University, Techn. Univ. of Delft, Delft Hydraulics, Francius Institute and WES, USA resulted in formulae for a number of broken units. References are:

Dolosse (includes roundheads): Burcharth et al. (1995).

Tetrapods: Burcharth et al. (1995).

In order to estimate the breakage by use of the formulae it is necessary to know the **strength of the concrete in the prototype armour units**. For this reason a field study of in-situ concrete strength in Tetrapods and Dolosse in Italy was undertaken in project (iii), Franco et al. (1995). Average tensile strength strength varied between 3.5 and 4.3 MPa and the coefficient of variation was app. 20%. WES, Vicksburg, USA has also made in-situ strength studies in the USA, but investigations from more countries are wanted.

Thermal stresses in concrete armour units due to heat of hydration developing during curing can weaken the units significantly, Burcharth (1983).

A field study of reduction of thermal stresses by introducing a hole in cubes is presented in Burcharth (1991).

In project (iii) a systematic study of curing induced thermal stresses in cubes and Tetrapods was performed by the use of numerical calculations, Nooru–Mohamed (1994). Recommendations of how to avoid critical stresses are given in the references.

Deterioration of concrete armour units due to solar stress was as part of project (iii) studied by field investigations in Italy and by numerical calculations, Canarozzi et al. (1995). It was shown that solar stress can be responsible for degradation of the concrete.

There is a potential risk of significant concrete strength reduction due to **fatigue effect** caused by repeated wave loading, Burcharth (1984). A systematic desk study using various fatigue models was undertaken in project (iii), and it was shown that fatigue strength reduction is critical if the in–situ tensile strength of the concrete is less than app. 2.5 MPa, Soerensen et al. (1995).

7.3 Reshaping breakwaters

The so–called berm breakwaters were studied in project (v). Main emphasis was put on profile development and stone transport at trunk and roundhead sections in oblique waves. Recent references are: Lamberti et al. (1994), van der Meer et al. (1995), Hald et al. (1995), Juhl et al. (1996), Alikhani et al. (1996), Hald et al. (1996). Based on these results as well as on earlier results, van der Meer (1988), it is possible to estimate the profile development, the threshold of stone movements, and the amount of stone transport.

7.4 Scour around roundheads

Formulae for scour around cone shaped and circular cylindric roundheads were developed from experimental studies at the Technical Univ. of Denmark as part of projects (ii) and (iii), Sumer et al. (1997), Fredsøe et al. (1997). Engineering tools for estimation of scour along breakwater trunks are not yet available.

8 Reliability of existing breakwaters

Level 2 reliability analyses of conventional deterministically designed rubble mound and caisson breakwaters were performed within the PIANC PTC II Workings Groups 12 and 28, projects (xiv) and (xv).

Large failure probabilities were found for rubble mound breakwaters; typically corresponding to 40–70% probability of severe damage within a 50 year life time, PIANC (1992), Ligteringen (1992).

For caisson structures designed according to Japanese recommendations, OCDI (1991), the safety level is higher but still very low compared to other civil engineering structures. Burcharth et al. (1995c), Christiani (1997), Ligteringen (1997). Further analyses will be made in project (vii).

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